

CHAPTER 1

INTRODUCTION AND LITERATURE REVIEW

1.1 Introduction

Vibrations occur in every floor system subjected to human occupancy. However, not all vibrations are annoying, since many are quickly damped out or are short lived. Some floor vibrations do cause complaints and irritability from occupants. In severe cases, the floor can become useless because nobody wants to be on it. Fixing floor problems is expensive, and some floors cannot be improved. The best remedy is to design floors that do not allow vibrations to become annoying. This solution requires a knowledge of how the components of a floor and the overall floor system behave when subjected to vibrations.

A commonly used structural element is the open web steel joist, which is very efficient structurally and is economical. It is therefore quite popular for steel framed buildings. However, the vibration of a steel joist is not as well understood other structural elements, such as hot rolled steel beams. Further study into their behavior can provide the knowledge which can allow floors using steel joists to be designed with more confidence that they will not be annoying to occupants of the buildings.

1.1.1 Scope of Research

The goal of this research is to develop a computer modeling technique which will accurately predict the fundamental natural frequency of open web steel joist supported floors with a concrete slab. To accomplish this goal, the study is divided into three sections. The first section is to evaluate previous methods used to model floors. Comparisons are made between the computer models, hand calculations and experimental results to determine the best model for a single open web steel joist-concrete slab tee beam. The next section uses the model selected above to investigate the current proposed

equations for the effective moment of inertia of a composite open web steel joist tee beam. Finally, a full single bay floor model is developed using information obtained in the above sections. This model is used to predict the fundamental natural frequency of seven existing floors to determine the effectiveness of the model.

1.1.2 Organization of the Study

First, the basic terminology used to discuss various principles of structural vibrations are introduced. Next, a discussion of previous research introduces the reader to the background used. Chapter 2 deals with the evaluation of current computer modeling techniques. Chapter 3 discusses the investigation of the proposed equations for effective moment of inertia of a steel joist - concrete slab system. Chapter 4 describes the development and testing of a full bay floor model. Finally, Chapter 5 provides conclusions and recommendations for future research.

1.2 Terminology

Vibration is the oscillation of an elastic system around its equilibrium position due to a sudden release from a deflected position or from the application of an impulse load. A *cycle* of vibration is the motion completed each time the system passes through the same point and is moving in the same direction. The time it takes for one cycle to occur is called the *period* of vibration, denoted by the symbol T . The *amplitude* of a vibration is the magnitude of the maximum displacement of the system from its equilibrium position. Figure 1.1 shows the definition of amplitude and period.

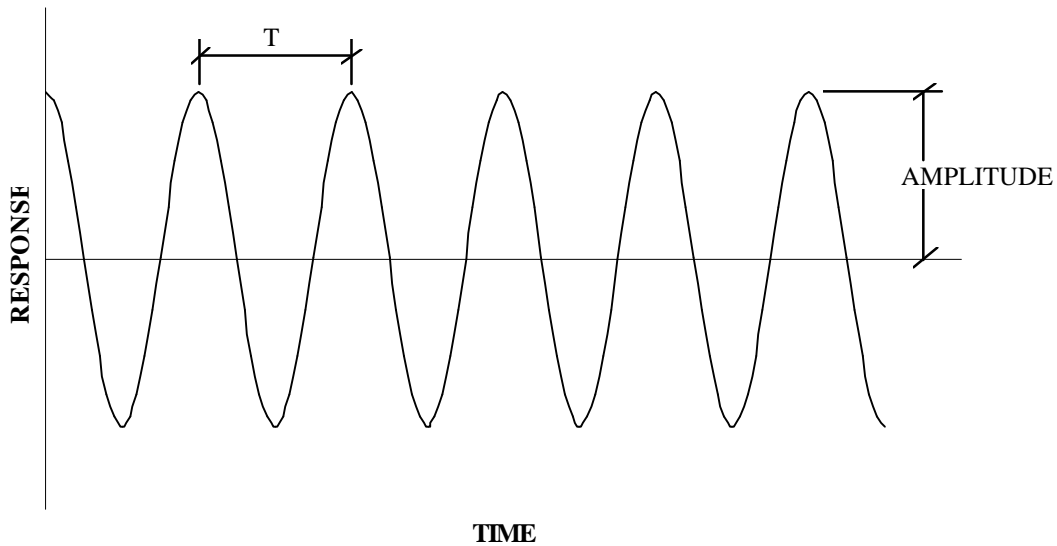


Figure 1.1 Definition of Amplitude and Period

Frequency is the number of cycles a system completes in one second, denoted by the symbol f . Frequency is the inverse of the period. The units of frequency are cycles per second, or Hertz (Hz). If a system is displaced and quickly released, or subjected to an impact load, the structure will vibrate. All structures have many natural frequencies, with the lowest one defined as the *fundamental natural frequency*, f_n . If a forcing function acts at or near the same frequency as one of the natural frequencies of the system, *resonance* occurs. This can result in very large displacements which can cause structural damage or failure if left unchecked. The *mode shape* of a system is the configuration of the structure during motion at a particular natural frequency. Every natural frequency has a corresponding mode shape.

Damping is the loss of mechanical energy during the vibration of a system. This energy loss results in a decaying motion, or decrease in amplitude, until the system ultimately comes to rest at its original equilibrium position. Figure 1.2 shows the effect of damping on the response of a vibrating system. Damping is usually modeled as *viscous damping*, which relates damping to the velocity of the system. This type of

damping is the easiest to model mathematically and generally agrees with experimental results. The damping required to prevent any oscillations of the system is called the *critical damping* of the structure. Usually, damping is given as the *damping ratio*. This is the ratio of the damping present in the system to the critical damping. The damping ratio is expressed as a percent of critical damping, and in most floors this value is between 1% and 5%.

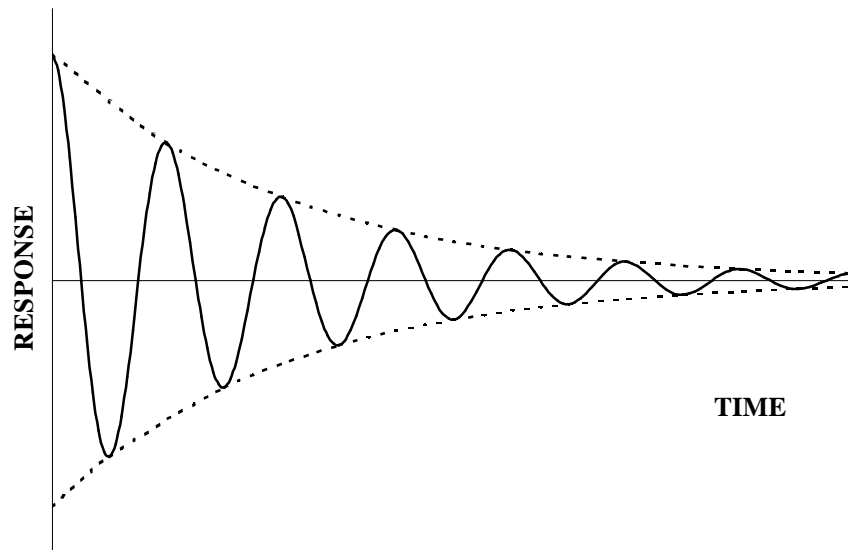


Figure 1.2 Effect of Damping on Response

1.3 Literature Review

1.3.1 Floor Frequency Equations

Steel frame - concrete slab floor construction has been used for over a century. Traditionally, a stiffness criterion was used to limit the live load deflection of beams or girders supporting plastered ceilings to span/360. Also, the span-to-depth ratio of the member was limited to 24 or less. These two criteria were widely applied to steel framed floor systems to control vibrations, but with little success (Murray, et al. 1997). A large change occurred in the steel construction industry with the widespread use of open web

steel joists, lightweight concrete, and longer spans. These changes in steel floor construction, while economically successful, increased the chance and severity of a floor vibration problem occurring. Also, these floors were not behaving as predicted by then current vibration criteria and design models. Therefore, over the past 30 years, much research has gone into understanding and developing more accurate floor vibration prediction equations and criteria to reflect modern designs.

The first basic equation in floor vibrations is the classical expression for the first fundamental frequency of a simply supported, prismatic, uniformly loaded, beam:

$$f_1 = 1.57 \sqrt{\frac{gEI}{wL^4}} \quad (1.1)$$

where g = acceleration of gravity = 386.4 in/sec²; E = modulus of elasticity, ksi; I = moment of inertia, in⁴; w = weight supported by the beam per unit length, kips/in; and L = span length of the beam. In using this expression to calculate the first fundamental frequency of a floor tee-beam, a knowledge of how the terms in Equation (1.1) are calculated is needed.

First, to model the effect of the concrete slab with the beam, a tee-beam model is used. For the purpose of frequency calculations, the slab is considered to act compositely with the beam, no matter what type of construction. Also, the dynamic modulus of elasticity of the concrete in the slab is used, which is 1.35 times the static modulus of elasticity E_c (Murray, et al. 1997). The effective width of the slab is taken the beam spacing, S , with a limitation of four-tenths of the length of the beam, $0.4L$ (Murray, et al. 1997). The effective depth of the slab, d_e , parallel to the beam is taken as the depth of concrete above the steel deck ribs (Rahman and Murray 1974). The width of the concrete section is transformed by the modular ratio to obtain an effective width of steel, b_{tr} , which is then used to calculate the transformed moment of inertia, I_t . Figure 1.3 shows the tee-beam model used to calculate the transformed moment of inertia which is then used in Equation (1.1).

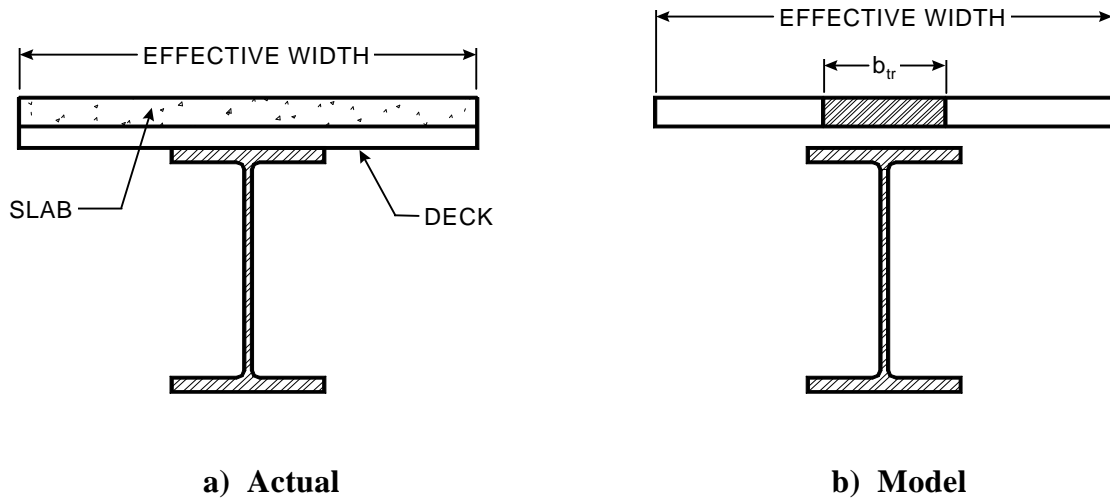


Figure 1.3 Tee-Beam Model

To determine the first fundamental frequency of a beam-girder floor system, Equation (1.1) is first used to calculate the frequencies of the beam and the girder. These are then combined using a relation called Dunkerly's formula given by

$$\frac{1}{f_s^2} = \frac{1}{f_b^2} + \frac{1}{f_g^2} \quad (1.2)$$

where f_s = the system frequency, Hz; f_b = the beam frequency, Hz; and f_g = the girder frequency, Hz. Equations (1.1) and (1.2) represent the basic equations in determining floor frequencies and are incorporated into most vibration criteria in some form.

Currently, the main problem is in accurately predicting the first fundamental frequency of open web steel joist supported floors. The tee-beam model works well for prismatic beam sections, such as a hot rolled wide flange steel beam. In a hot rolled beam, the flanges generate most of the stiffness of the section, with the web acting as a separator for the flanges. The web also resists the transverse shear that a beam experiences under flexure. The additional deflection caused by transverse shearing is usually assumed to be negligible in a wide flange section compared to the deflection caused by flexure. Only in very deep, short beams does shear deformation become noticeable.

An open web steel joist also derives most of its stiffness from its flanges, or chord members. The web of a steel joist is composed of many smaller members, usually light angles or round bars. Like a hot rolled beam, these members keep the chords separated; however, these members resist transverse shear as an axial load. The stiffness of the web members and their resistance to buckling are important factors in the overall bending resistance of the steel joist. When web members experience large axial deformations, additional vertical deflection occurs which is referred to as shear deformation.

For the above reasons, it has long been known that using the full moment of inertia of the chords overestimates the actual bending stiffness of a steel joist. The Steel Joist Institute has long used a factor of 0.85 to reduce the moment of inertia of the chords when determining the deflection of joists (SJI 1995). This approach is generally sufficient for span-to-depth ratios, L/D , greater than about 18. Below this value, significant error occurs (Kitterman 1994). Studies have recently been completed to develop more accurate models of the effective moment of inertia of steel joists.

Kitterman (1994) did a study using 25 different joist and joist girder designs with varying span-to-depth ratios. He modeled the joists using the finite element program SAP90 (Wilson and Habibullah 1992), subjecting them to a uniform loading. He obtained the computed maximum centerline deflection and then back calculated the moment of inertia for the joist. He then compared this to the full moment of inertia of the chords. Using this data, he developed the following empirical equation for determining the effective moment of inertia of a steel joist:

$$I_{eff} = [0.6245 + 0.0084(L / D)]I_{chords} \quad (1.3)$$

where L is the span of the joist and D is the depth of the joist. This study involved joists having a span-to-depth ratio ranging from 10 to 24. This equation states that the stiffness of the joist is dependent on its span-to-depth ratio. For a span-to-depth ratio of 10, the modification factor from Equation (1.3) is 0.7; for a span-to-depth ratio of 24 it is 0.83.

Band (1996) expanded on Kitterman's work to include a larger range of span-to-depth ratios of 2 to 24. The reason for the lower span-to-depth ratios was to incorporate

the ranges for common joist-girders. He also increased the total number of joist designs, and separated round bar web designs from angle web designs. The separation of angle web joists from round bar web joists was made when Kitterman (1994) noticed that the round bar web data points did not correlate well with the rest of the data. Band (1996) then separated the two web types and developed independent equations for both. Using the same finite element program and the same procedures as Kitterman used, he developed the following two equations from his data:

$$I_{eff} = [0.8455(1 - e^{-0.28(L/D)})^{2.8}]I_{chords} \quad (1.4)$$

$$I_{eff} = [0.721 + 0.00725(L/D)]I_{chords} \quad (1.5)$$

Equation (1.4) is for angle web joists and Equation (1.5) is for round bar web joists. Band's recommendations were incorporated into the Design Guide "Floor Vibrations Due to Human Activity" (Murray et al 1997). Some slight modifications were made, and provisions for incorporating the composite moment of inertia and the effect of joist seats on girder stiffness were included. The following series of equations are currently recommended in the Guide for use when calculating floor vibrations of steel joist floors:

For bare joists:

$$I_{mod} = C_r I_{chords} \quad (1.6)$$

where:

I_{chords} = moment of inertia of the joist chords, in⁴

C_r = $0.9(1 - e^{-0.28(L/D)})^{2.8}$ for angle web joists ($6 \leq L/D \leq 24$)

= $0.721 + 0.00725(L/D)$ for round bar web joists ($10 \leq L/D \leq 24$)

Note that the coefficient of 0.8455 in Equation (1.4) is changed to 0.9 in Equation (1.6). The reason for this change is due to the way joists are manufactured. Band (1996) used nominal angle sizes when modeling the joists. Actual manufactured joists have slightly larger chord sizes than those specified in the design of the joist. This creates a stiffer

joist, which is reflected in the increased constant. For composite joists, the Guide recommends

$$I_{eff} = \frac{1}{\frac{g}{I_{chords}} + \frac{1}{I_{comp}}} \quad (1.7)$$

where:

$$g = \frac{1}{C_r} - 1 \quad (1.8)$$

and

I_{comp} = the transformed moment of inertia using the full chord areas

Equation (1.7) comes from a derivation which uses the deflection of a beam due to shear and flexure and relates this to the effective composite moment of inertia. This derivation is found in Appendix A.

The effect joist seats have on the composite moment of inertia of a girder has been studied over the past several years (Murray, et al. 1997). It was found that the composite moment of inertia calculated for girders with joist seats was higher than the moment of inertia determined from tests. One explanation for this behavior is that the joist seats do not provide enough shear transfer between the girder and the slab to justify using the fully composite moment of inertia. Therefore, Equation (1.9) is recommended in the Guide when calculating the moment of inertia of girders or joist girders with joist seats:

$$I_g = I_{nc} + (I_c - I_{nc}) / 4 \quad (1.9)$$

where:

I_{nc} = non-composite moment of inertia of the girder (if this is a steel joist girder, use I_{mod} in place of I_{nc} .)

I_c = composite moment of inertia of the girder (if this is a steel joist girder, use I_{eff} in place of I_c .)

1.3.2 Survey of Various Computer Modeling Techniques for Steel Joists

Computer modeling of structures has become very sophisticated with the development of powerful computers. Complex structures can be analyzed quickly and easily with desktop computers. Structural analysis programs are easier to use, as many switch from a text based to a graphical based user interface. However, even with the ease of programming and running an analysis today, knowledge of how a structural system behaves within the computer model is a must. Computer models require many assumptions as to how the real structure might behave. The most basic assumption is that a finite number of degrees of freedom will model an elastic system that has an infinite number of degrees of freedom. The model may also have to be simplified so that it can fit within the confines of the particular software or hardware constraints. This section will review the computer modeling techniques used for steel joist-concrete slab floor vibration models.

The first type of beam-slab model is the in-plane model of a tee-beam. Here, the slab and the beam share a common center of gravity, in a horizontal plane. The frame elements share common nodes with the slab elements, which creates a very simple model with relatively few nodes, elements and degrees of freedom to describe the system. The total stiffness of this model is derived from the sum of the stiffnesses of the beam and the slab. The user inputs the moment of inertia of the beam as the total transformed moment of inertia of the single tee-beam, minus the transformed moment of inertia of the slab alone (Kitterman 1994). The main advantage of this model is the small number of nodes and elements needed to define a joist-slab system. Also, modeling large numbers of beam-slab configurations is easy with this setup. For a given slab configuration, any beam section can be modeled by simply changing the moment of inertia of the frame element. Figure 1.4 shows this type of model.

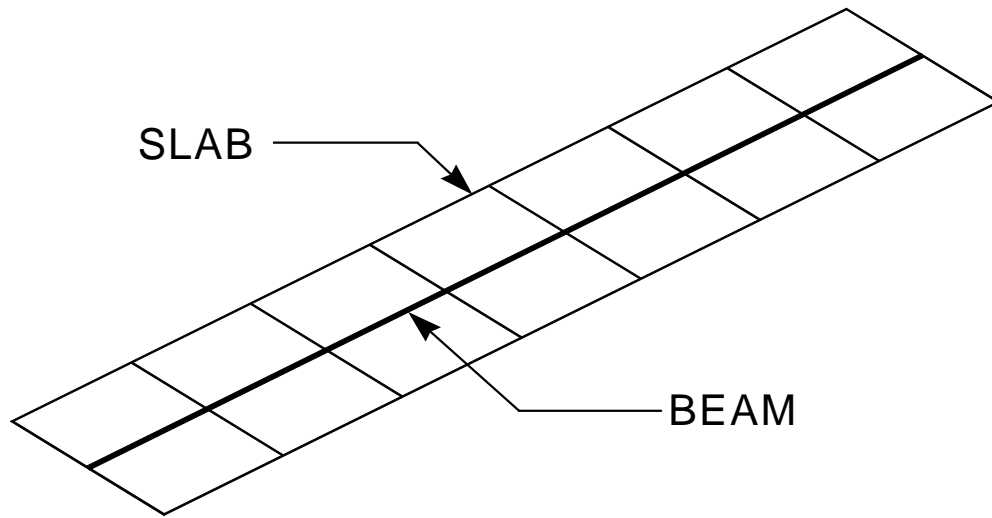


Figure 1.4 In-Plane Computer Modeling Method

Rottmann (1996) used this type of modeling technique to describe several test floors and floors in office buildings. The purpose of the models was to determine the frequencies and mode shapes of the first several modes. All of the floors had steel joists for the floor beams which were supported by either walls or hot rolled girders. The beams and girders were all modeled as frame elements and placed in the same plane as the slab. The slabs were modeled using plate elements. Enough plate elements were used to develop a sufficiently fine mesh to get acceptable results. Most plate elements had an aspect ratio of 1 to 1. The first floor that Rottmann modeled was an experimental floor Hanagan (1994) had built. Since the frequencies of the floor were known, the model could be adjusted to match that of the real floor. Hanagan had discovered that the supports of the floor were moving vertically, so springs were used to model the supports and bring the frequencies closer to those observed. Rottmann used similar modeling techniques for the other floors she modeled which were parts of office buildings. Some of these techniques included using non-composite moments of inertia, rotational springs on the edges of the slabs, and inclusion of a portion of the live load as mass in the system. Each of these produced varying results, and the one which gave the closest frequencies and mode shapes to those observed was used.

The next basic type of model is to place the beam at the elevation of its centroidal axis, the slab at the elevation of its centroidal axis, and then link them together with either rigid links or some kind of joint constraint. This increases the number of elements, nodes, and degrees of freedom of the system. However, it removes some of the hand calculation needed to define the moment of inertia of the beam. This model requires more effort to modify than the in-plane model, since the distance between the beam and the slab centroids varies with each design. Shamblin (1989) used this model for her 240 floor designs, which were composed of both hot-rolled and steel joist beams. Figure 1.5 shows this type of model.

The final type of model is the full joist model. This model is specific to steel joists and uses frame elements to model the entire truss of the joist. The slab is attached to the top chord of the joist by the use of rigid link elements. This model is the most complicated, since it requires knowledge of the dimensions and properties of every member of the joist, including the web members. The joist is modeled in one plane, and prevented from having displacements out of that plane. This model is useful in determining the behavior of a joist, such as its stiffness and deformation properties. Both Kitterman (1994) and Band (1996) used this type of model to determine the effective moment of inertia of steel joists, which lead to the development of Equations (1.3), (1.4), and (1.5). Figure 1.6 shows this type of model.

Unlike a true truss, where the members are connected by frictionless pins and cannot develop any moment, joist connections are welded and therefore can develop moments. Gibbings (1993) did a study on the best way to model the web-chord connections of a steel joist. He discovered that if all of the members in a web-chord connection shared a common node, the joist model was overly stiff. To correct this problem, Gibbings suggested using joint eccentricities to more accurately model the load path from the web member to the chord member. He recommended that two web members framing into the chord be separated by 2 in. If a vertical member is present, it should connect midway between the web members. Gibbings studied the effect of changing the length of the joint eccentricity from 0.5 in. to 3 in. He determined that there

was less than a four percent difference in the deflection of the joist due to the different joint eccentricity lengths. As long as some joint eccentricity is included, the model will more accurately predict the behavior of the joist. This technique allows the joist model to be less stiff, which correlates well with experimental tests. However, it does greatly increase the number of nodes and elements in the model. Band (1996) determined that for round bar webs, joint eccentricity was not needed since the web is a continuous round bar and the load path can be accurately modeled without using joint eccentricity. Figure 1.7 gives an illustration of joint eccentricities.

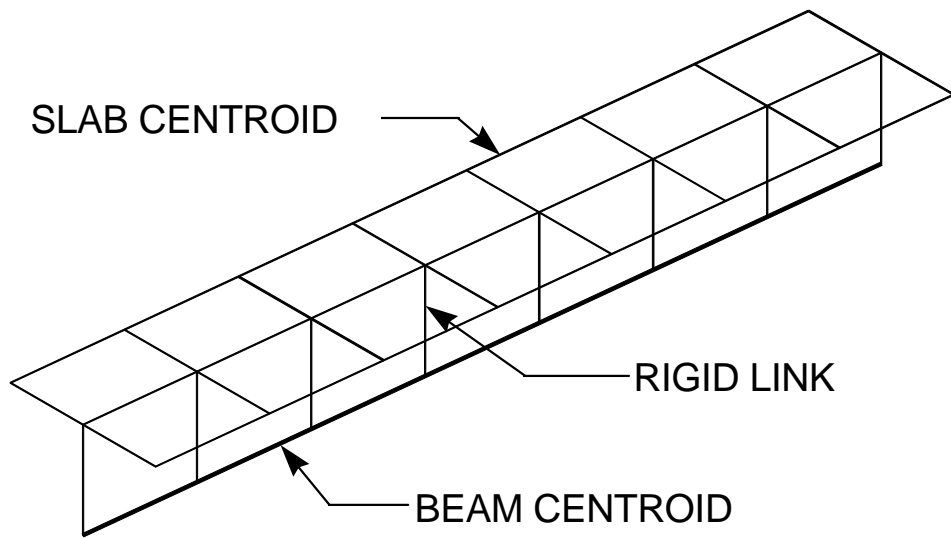


Figure 1.5 Beam to Shell Computer Model

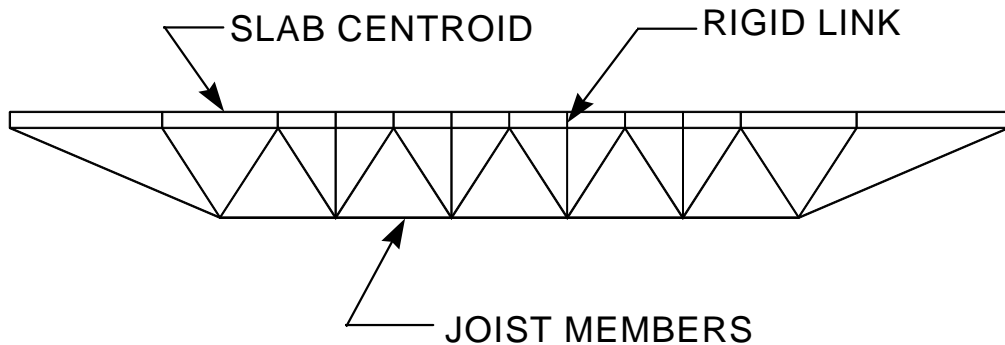


Figure 1.6 Full Joist Computer Model

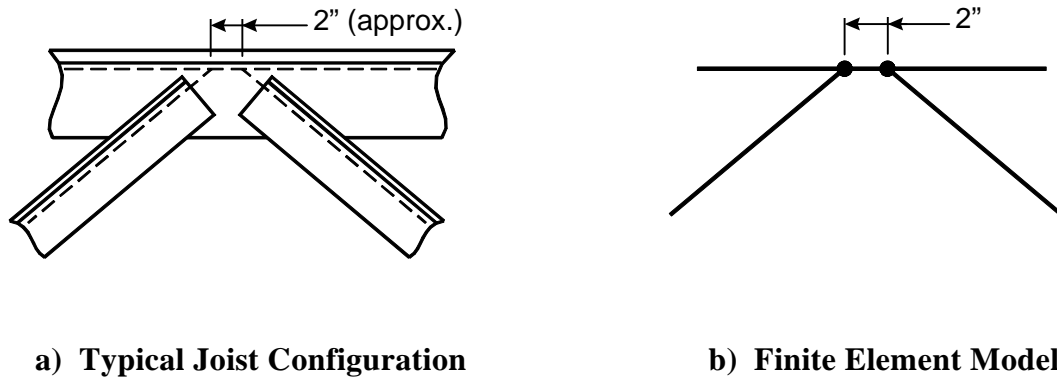


Figure 1.7 Definition of Joint Eccentricity

1.4 Need for Research

While much research has been done to determine the actual stiffness of steel joists and how to model these joists, there are still areas that require further research. The variety of ways to model steel joist-concrete slab systems is investigated to determine which model gives the most consistent prediction of the first natural frequency when compared to experimental results. Using the model chosen, the proposed equations for determining floor vibrations are investigated. The previous research on these equations dealt only with the joist, while this research looks at the joist-slab system and its agreement with the proposed equations.

Finally, a full bay floor modeling technique is developed that can be used to systematically determine the first fundamental frequency of a steel joist supported floor. Previous floor models had to be specifically tailored to match the behavior of the actual floor. Therefore, these models are poor in consistently predicting the frequency of floors before they are built. This fact was demonstrated by several floors that were built which vibrated at a frequency less than 10 Hz, while every model predicted a frequency greater than 10 Hz. The model developed must be able to predict a floor's behavior during design so that vibration problems can be avoided.